APPENDIX 8-9

TEST CELL PROGRAM TECHNICAL MEMORANDUM

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BLACK & VEATCH

TECHNICAL MEMORANDUM

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Task 16.02 Test Cell Program Technical Memorandum

To: Distribution

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1. INTRODUCTION

This technical memorandum describes the construction, testing, and results of the Everglades Agricultural Reservoir Test Cell Program completed in western Palm Beach County, FL during January through May of 2005. The program involved the design, construction, installation of instrumentation, watering, dewatering, and monitoring of two test cells and encompassing seepage canals. Each test cell measured 500 feet square (at the embankment centerline) and consisted of an impoundment enclosed by a zoned earthen embankment surrounded by a seepage collection canal (SCC). The test cell site is located within the footprint of the planned Everglades Agricultural Area Reservoir A-1 (EAA Reservoir A-1).

The Test Cell Program is part of the investigations program being completed for design of the EAA Reservoir A-1. The test cells were located and designed by Black & Veatch with authorization from the South Florida Water Management District in Work Order No. 2. The Test cells were constructed by Barnard Construction, Inc. (Bozeman, MT) with inspection provided by Black & Veatch. Construction subcontractors included HB Mellott Estate, Inc. (Warfordsburg, PA) for the rock processing, Angelini Blasting, Inc. (Fort Lauderdale, FL), INQUIP Associates, Inc. (McLean, VA) for Test Cell 2 soil-bentonite cutoff wall construction, Southern Power and Controls Corporation (Tampa, FL) for electronic instrumentation installation and setup, and Nodarse Associates, Inc. (West Palm Beach, FL) for exploratory drilling, piezometer installation, and construction laboratory testing. The test cell filling and testing was also conducted by Barnard Construction with direction and monitoring by Black & Veatch.

1.1 Test Cell Program Objectives

The objectives of the test cell program were to provide information for the Engineer to develop seepage modeling and embankment design criteria for the EAA Reservoir A-1. A detailed breakdown of the objectives is listed below:

- To evaluate the effectiveness of a foundation cutoff in controlling seepage from the cells by constructing two embankment designs, one with and one without a soil-bentonite cutoff wall

- To investigate the in-situ earth materials (local formations) and their use as embankment construction materials
- To investigate the foundation conditions and evaluate the foundation treatment necessary for embankment construction
- To gain practical experience handling and placing the local earth materials in construction of the embankments allowing the assessment of the requirements and techniques necessary to optimize construction of the EAA Reservoir A-1
- To obtain data to define the hydraulic conductivity of the local formations (less than 100 feet depth) to be used in estimating the total seepage from the EAA Reservoir A-1
- To obtain data on construction production rates and costs
- To evaluate the quality and production of erosion protection materials (riprap) from the Fort Thompson Formation limestone caprock

To evaluate the quality and processing requirements of crushed rock construction materials (embankment filter and drain and concrete aggregate) from the caprock at the top of the Fort Thompson Formation.

1.2 Report Organization

1.2.1 Report Text

The remainder of this section will present the background information on the test cells program, detailing the site location, geology, and previous investigations. The following report sections will detail the test cell design, recount the construction history with the construction methods and equipment, recount the testing program, evaluate the construction program highlighting insights and lessons learned, evaluate and analyze the testing program, and present important conclusions and recommendations. Data appended to this report as a compact disk include the logs of geotechnical borings completed for design of the test cells (Appendix 8-16), the logs of geotechnical borings completed during construction (Appendix 8-17), the Daily Construction Reports (Appendix 8-18) the Daily Construction Equipment Logs (Appendix 8-19), typical open standpipe piezometer installations (Appendix 8-20), test cell embankment and stockpile construction quality control testing data (Appendix 8-21), the test cell pumping and level data (Appendix 8-22), representative project photographs (Appendix 8-23) and triaxial testing results on select fill (Appendix 8-24).

1.2.2 Drawings

An 11-sheet set of drawings was prepared by Black & Veatch for the construction of the test cells. The following sheets were prepared and are attached to this report:

Sheet No.	Title/Description
1	Cover Sheet
2	Overall Site Plan
3	Typical Test Cell Plan
4	Typical Test Cell Cross Section
5	Seepage Collection Canal Pump Plan
6	Makeup Water Pump Plan
7	Trailer Area Plan
8	Sections and Details
9	Piezometer Sections and Details
10	Piping and Instrumentation Diagrams
11	Instrumentation Details

1.3 Background

1.3.1 Site Location and Characteristics

The Test Cell Program site and the planned EAA Reservoir A-1 are located in western Palm Beach County, FL in the Everglades physiographic area, an area of low relief with the site natural ground surface lying generally between elevations 8 and 11 feet (NAVD88). The test cell site is located in the southwest and southeast quarters of section 6 and the northwest and northeast quarters of section 7 of Township 46S and Range 37E. The location is shown on Sheet 1 of the Drawings.

The test cell site is an agricultural area primarily used for the growing of sugar cane. The area is drained by a system of canals constructed during the second half of the last century as the Central and Southern Flood Control Project. Pumping and flooding of these canals is used by the local sugar producers to regulate groundwater level for planting and harvesting of the crops. It has a subtropical climate with between 55 and 63 inches of rain per year, primarily during June through October.

The test cell site layout included a site access road system, a field office trailer area, the two test cells with associated seepage collection canals (SCC), and an on-site riprap test blast/borrow area.

1.3.2 Previous Investigations

One hundred forty-five geotechnical borings were completed for the South Florida Water Management District (District) around the planned EAA Reservoir A-1 in 2003 and early 2004. Four of those borings are located in the vicinity of the test cell site: CB0068, CB0069, CB0140, and CB0142. Boring CB0068 is about 800 feet northwest of the test cell site Borrow Area. Boring CB0069 is located over 1000 feet west of Test Cell 1. Boring CB0140 is located about 800 feet east of Test Cell 2. Boring CB0142 is located about 200 feet east of the Borrow Area and 1500 feet north of the test cells. The borings were completed with rotary wash drilling and standard penetration testing. They were between 50.5 and 52 feet deep.

1.3.3 December 2004 Investigations

Twenty geotechnical borings were completed at the Test Cell Program site in December of 2004, including 10 at the site Borrow Area and five at each test cell. The borings were all drilled to a depth of 50 feet, primarily by rotary wash drilling using a heavy drilling mud to support the holes. The near surface limestone (caprock) was cored in each one of the holes, and a deeper, thinner limestone was cored at about 26 feet bgs in two of the borings. Soils were sampled with standard penetration tests. Twenty samples of the limestone cores were tested for unconfined compressive strength (ASTM D2938). Selected samples of the soils were tested for gradation (ASTM D422), moisture content (ASTM D2216), carbonate content (Florida Test Method Designation FM 5-514), and percent passing the 200 sieve (ASTM D1140).

The boring logs are contained in Appendix 8-16. The rock testing and soil testing results are summarized in Tables 1 and 2, respectively.

1.3.4 Area and Site Geology

The land surface at the test cell site and the entire site of the EAA Reservoir A-1 is covered with a 1 to 2 feet thick, black, highly organic, fine grained soil known locally as muck or peat. The peat is often underlain by several inches to 2 feet of calcareous clay locally called marl. The peat and marl constitute the local soil in the entire Everglades and EAA Reservoir A-1 area.

Schroeder et al. (1954) indicated that the geologic formations underlying this soil comprise two aquifers, including a near surface (surficial) water table aquifer separated from a deeper artesian aquifer by a relatively impermeable series of strata constituting an aquitard. These aquifers are the surficial aquifer system and Floridian aquifer system:

- The Fort Thompson Formation, underlying Caloosahatchee Formation and upper portion of the Tamiami Formation are included in the surficial aquifer system.
- The deeper artesian Floridian aquifer system is composed primarily of limestone belonging to the Ocala Limestone, and Avon Park and Oldsmar Formations.
- The intervening impermeable strata belong to the Tamiami Formation and the upper part of the Hawthorn Group.

According to Schroeder et al., the Fort Thompson Formation is composed of limestone, carbonate sand and silty sand. The Caloosahatchee Formation composed is primarily of quartz and carbonate sand, with minor amounts of limestone. In the western Palm Beach County Everglades area it thins from about 70 feet thick at Belle Glade to about 7 feet near the Broward County line. The Tamiami Formation is primarily sand, clayey sand and poorly consolidated, sandy limestone. It is 70 to 100 feet thick in west Palm Beach

County, thinning eastward. The upper part of the Hawthorne Group is primarily calcareous sand and clay. The surficial aquifer system as presented in Schroeder et al. (1954) is generally confirmed in Scott (1977) to be approximately 250 feet.

The borings completed at the test cell site in December 2004 prior to the Test Cell Program and during construction penetrated through the soil, then through 22 to 26 feet of primarily carbonate sand and limestone, and then into primarily shelly quartz sand with sparse limestone to their full depths at 50 to 100 feet. The upper carbonate sand and limestone constitutes the Fort Thompson Formation at the site. Below this the shelly sand with sparse limestone constitutes the Caloosahatchee Formation and possibly part of the Tamiami Formation

The site soil consisted of about 1 to 2 feet of peat, locally overlying up to several inches of marl. Immediately below the site soil, the top of the Fort Thompson is a hard limestone layer about 4.5 to 5 feet thick, locally called caprock. The caprock and the overlying marl are fresh water deposits known as the Lake Flirt Marl. The caprock is underlain primarily by silty carbonate sand down to about 23.5 to 24.5 feet depth where another hard limestone layer, 1.5 to 3 feet thick, is encountered. A thinner, hard limestone layer, about ½ to 1 foot thick, is often encountered at around 16 to 17 feet depth. Laboratory testing on the sand sampled in the borings averaged 84.2% calcite content with an average of 22% passing the #200 sieve in gradation tests. Visual inspection of the sand samples from the borings reveals that it consists at least partly of shell fragments, and tends to be angular and platy.

All the limestone layers exposed in core or in excavations are very fossiliferous. The sand exposed in the SCCs and site dewatering sumps was abundantly fossiliferous with gastropods, pelecypods, corals, and echinoderms. The caprock is white, light gray, tan and yellowish brown. The sand and lower limestone layers are white to very pale brown.

The limestone layers in the Fort Thompson Formation appeared to be the primary sources of groundwater seepage into the site excavations made during the construction of the test cells. Water could be seen streaming from the bottom of each of the three limestone layers in the dewatered excavations. They are all fractured, and the caprock contains solution cavities including local areas of interconnecting channels (especially near the top) and single channels up to several inches in diameter that penetrate the full thickness. The solution channels in the caprock locally contain soil including the peat/muck and marl.

The shelly quartz sand encountered below the bottom limestone of the Fort Thompson is fine grained, subrounded, quartz sand that is mixed with shelly carbonate sand similar to that in the Fort Thompson. The proportions of carbonate to quartz sand vary greatly. Also, only a few short intervals of hard drilling less than a foot thick were encountered in any of the borings at these depths. Laboratory testing on the sampled sand gave an average calcite content of 40.1% and an average 12.1% passing the #200 sieve. The material color also changes to a light greenish gray at some depth below the bottom Fort Thompson limestone bed. The materials are typical of the Caloosahatchee Formation.

If the formation thicknesses given in Schroeder et al. are accurate, the deeper borings at the Test cell site should have penetrated into the Tamiami Formation. However, no silty or clayey strata were encountered in any of the borings, even those to 100 feet depth. Similarly, borings up to 110 feet deep at the STA-3/4 and East WCA-3A Hydropattern Restoration south of the EAA Reservoir A-1 and test cell sites did not encounter silty or clayey strata (Nodarse & Associates, 2001). These borings could have penetrated into the Tamiami with no appreciable change in appearance though.

More recent geological work (Reese and Cunningham, 1998) has redefined the stratigraphy of the area. Presently, the Tamiami is recognized to have two members, the Ochopee Limestone and the Pinecrest Sand, and the clayey sand units formerly in the Tamiami Formation have been assigned to a new formation, the Peace River Formation in the Hawthorn Group. Both Tamiami members contain sandy strata, but the Pinecrest Member is principally shelly, fine grained, quartz sand. The Caloosahatchee and Tamiami Formation sands are generally differentiated in exposure on the basis of fossils, but this is not possible in borings (personal communication from Thomas M. Scott, Ph.D., P.G., and Assistant State Geologist). For this reason, it is probably best to refer to the shelly quartz sand encountered in the borings below the Forth Thompson at the Test cell site as the Caloosahatchee/Tamiami Formation.

Furthermore, the bottom of the surficial aquifer system in the area of the Test Cell Program and EAA Reservoir A-1 may be much deeper than any of the borings completed to date in the area. Miller (1987) contoured the bottom of the surficial aquifer system (the top of the Hawthorn Group) in Palm Beach County using existing well logs. According to this work the bottom of the surficial aquifer system in the area of the Test Cell Program and The EAA Reservoir A-1 lies between about elevations -200 to -220 feet.

2. TEST CELL DESIGN

2.1 General Layout

Sheet 2 of the Drawings shows the layout of the test cells and Borrow Area, and Sheet 3 shows the typical test cell plan. Each test cell consisted of a square area enclosed by a zoned earthen embankment and surrounded by an excavated and unlined SCC. The embankments and SCCs are separated by a bench, a level stripped area. Each SCC was equipped with a pump or pumps and piping to return collected seepage to the test cell.

An existing road and irrigation canal (the Primary Canal) are located along the southern edge of the test cell site. Water to fill the cells and makeup water to compensate for seepage bypassing the SCCs was supplied from the Primary Canal by pumps installed just north of the existing road. The temporary piping to convey the water to the cells paralleled the site access road constructed from the existing road to the cells.

2.1.1 Test Cell 1

Test Cell 1 (TC1) embankment was designed without a foundation seepage soil-bentonite cutoff wall. It was constructed directly on cleaned caprock. Typical sections for the original designs of both test cell embankments and SCCs are shown on Sheet 4 of the Drawings.

The embankment upstream slope was designed with a layer of large stone riprap for erosion protection. This was separated from the upstream, sloping embankment core by a thin layer of gravel bedding. A sloping chimney filter and drain separated the core material designated as select fill, from a zone of random fill to the downstream. The sloping chimney filter and drain extended as blankets under the random fill zone to the outer rock fill zone. A 10-foot wide berm of select fill placed around the embankment downstream toe retained seepage in the rock fill for collection into two drain sumps installed in the eastern and western downstream toes of the test cell embankment.

The drain sumps were formed from a pre-cast concrete pipe placed on a layer of granular fill. Short lengths of perforated HDPE pipe extended from the sumps into the rock fill to collect seepage released from the filter and drain under the random fill zone. Float operated pumps contained in the sumps returned the seepage to the SCC.

Each embankment zone was designed to utilize locally excavated material, primarily from the SCC excavations. The zone materials were identical for both embankments. All were required to be free from organics. The following are the general specified characteristics for each embankment zone:

- Select fill was defined as well-graded, minus 4-inch material
- Random fill was defined as 12-inch minus material
- Rockfill was defined as clean, well-graded rock less than 24 inches in maximum dimension and containing less than 10 percent by weight passing the 1-inch screen
- Filter material was defined as well-graded processed material with 100 percent, 90 to 100 percent, 20 to 55 percent, 5 to 30 percent, 0 to 10 percent, and 0 to 5 percent passing the ½-inch, 3/8-inch, No. 4, No. 8, No. 16, and No. 50 sieves, respectively
- Drain material was defined as well graded processed material with 100 percent, 95 to 100 percent, 35 to 70 percent, 10 to 30 percent, and 0 to 5 percent passing the 2-inch, 1 ½-inch, ¾-inch, 3/8-inch, and No. 4 sieves, respectively
- Riprap was defined as reasonably well graded rock pieces with a median size of 24 inches and a maximum size of 36 inches

The SCC design section for both test cells was identical as shown on Sheet 4 of the Drawings. The canals were unlined and crossed only by the site access road. At TC1 the area under the access road crossing the canal was not excavated. At Test Cell 2 (TC2) it was excavated and then backfilled with rockfill to form a base for the road.

An approximately 120-foot square area was shot in the foundation in the center of each test cell. This was done to breach the caprock and enhance seepage access to the foundation, approximating a worst-case flow condition. Breach of the caprock in the test cells added a discontinuity in the model which allowed the properties of the caprock and Fort Thompson to be distinguished; it did not eliminate the caprock from the assessment. In TC1 the shot rock was left in place. In TC2, Barnard partially excavated it as a source of rockfill.

2.1.2 Test Cell 2

Test Cell 2 (TC2) embankment was designed with a select fill central core and a soil-bentonite cutoff wall extending through the bottom of the core into the foundation. This is buttressed upstream by a zone of random fill and a rockfill zone for erosion protection. A chimney filter separates the core from a downstream random fill zone. The chimney filter continues downstream from the chimney as a blanket overlying a blanket drain. Both blankets extend downstream under most of the downstream random fill zone and extend the remainder of the way to an embankment toe drain as fingers. A PVC pipe perforated at its lower end, placed in each finger drain, and rising out of the fill allows monitoring of the drain water level.

The downstream toe drain contains a perforated HDPE pipe around the entire embankment perimeter that drains to two toe drain sumps located at the eastern embankment corners which are the lowest. A layer of select fill placed around and covering the toe drain contains the seepage in the drain. The sumps are formed from a pre-cast concrete pipe placed on a concrete slab. Float operated pumps contained in the sumps return the seepage to the SCC.

2.1.3 Test Cell Instrumentation

Each test cell has two nests of three open standpipe piezometers installed midway along each side in the bench between the embankments and the SCCs, one nest near the embankment downstream toe and one nest near the SCC. In TC1 the nests are aligned perpendicular to the embankment and SCC (opposite to what is shown on Sheet 3 of the Drawings) and separated by about 10 feet to allow room for drilling the installation holes. Each nest contains a shallow piezometer to about 25 feet depth, a mid level piezometer to 60 feet depth, and a deep piezometer to about 100 feet depth. They are all screened at the bottom with a 10-foot screen in the shallow piezometer and 20-foot screens in the deeper piezometers. The arrangement of each nest places the shallowest piezometer closest to the embankment and the deepest next to the SCC. Typical open standpipe piezometer installations are shown in Appendix 8-20.

TC1 also has two other open standpipe piezometers installed in the foundation beneath the embankment on all four sides, both with 5-foot screens. One installed 20 feet downstream from the embankment centerline (the embankment piezometers) was drilled 15 feet into the foundation before placement and extended up as the embankment was constructed. The second (the foundation piezometers), 30 feet upstream from the centerline, was installed by drilling through the finished embankment and 5 feet into the foundation. In addition to the open standpipe piezometers, a vibrating wire piezometer was installed at a depth of 10 feet into the embankment core on each side, 26 feet upstream from the centerline.

In TC2 the bench piezometer nests are similar to those in TC1, but aligned parallel to the embankment and SCCs as shown on Sheet 3 of the Drawings. They are again spaced about 10 feet apart for working room. TC2 also has embankment open standpipe piezometers on each side, 20 feet downstream of the centerline, and vibrating wire piezometers installed in the core select fill at the centerline.

Each test cell has staff gages and electronic pressure transducers to read water elevation in the cell and in the SCC. Flow meters are installed in the seepage return and fill/make-up water piping to the cells and also the drain sump pump discharges. These flow meters readout locally and electronically. The electronic readouts for the water elevations and flow meters are all routed to a Remote Terminal Unit (RTU), one located at each test cell, containing an automatic data logger, a circular chart recorder, and LED displays. Sheets 10 and 11 of the Drawings illustrate this level and flow instrumentation.

In addition to the open standpipe piezometers installed in the test cells, 3 nests of open standpipe background piezometers were installed outside the test cells, one nest southwest of TC1, one nest northeast of TC2, and one nest between the test cells. The locations of these background piezometers are shown on Sheet 2 of the Drawings. The nest construction is similar to those installed on the test cell benches, and they are aligned north to south with the deepest at the north end. Also, during the testing and monitoring period two more shallow piezometers were added, each about 500 feet south of TC2 SCC corners. All the open standpipe piezometers except for these last two contain automatic electronic data loggers (In-Situ, Inc. miniTrolls).

2.1.4 Test Cell Makeup Water and SCC Recirculation

The original design of the test cell makeup (fill) water and SCC recirculation systems (Sheets 5 and 6 of the Drawings) required two diesel hydraulic, submersible pumps, a 12-inch and an 8-inch, for the makeup water and one 8-inch pump at each test cell to recirculate seepage water from the SCCs to the cells. Comparable diesel powered centrifugal pumps with vacuum suction were installed in lieu of submersible pumps.

The makeup water pumps drew irrigation water from the Primary Canal along the south side of the site and were provided with piping and valves to allow delivery to either or both test cells. Each seepage recirculation pump drew water from the respective SCC and discharged to the test cell. The makeup and recirculation discharges at each test cell were monitored by flow meters that read-out directly and electronically to the RTU.

2.2 Modifications during Construction

During construction, TC1 embankment section was narrowed to speed completion. The random fill above the filter and drain blankets was eliminated in the design and replaced by rockfill. This became the downstream zone in the dam. The downstream embankment slope was also increased to 2:1(hor/vert). The resulting crest width was reduced from about 67 to about 30 feet. Figure 1 shows the modified section.

3. CONSTRUCTION HISTORY AND METHODS

3.1 General Sequence

Barnard Construction began mobilizing to the test cell site during the week of January 10 through 16. All major construction equipment except that for construction of the soil-bentonite cutoff wall in TC2 was on site by February 3. The rock processing plant components were delivered to the site and assembled between January 25 and February 1. Black & Veatch monitored the construction activities and prepared Daily Construction Reports and Daily Construction Equipment logs. The Daily Construction Reports and the Daily Construction Equipment Logs are contained in Appendices 3 and 4.

The Borrow Area was stripped of the surface peat and marl first; an initial blast for excavation was made and dewatering sumps installed. Work then shifted primarily to the test cell construction. The peat was stripped from the test cell areas, first TC2 and then TC1. SCC excavation began in TC2 first as well as embankment construction. However, construction of TC1 embankment continued during the installation of the soil-bentonite cutoff wall in TC2, consequently TC1 SCC and embankment were completed first.

Material was generally not taken from the Borrow Area for actual test cell construction. An initial shot was made early in the construction period using 10 and 20 feet depth shot holes on January 18 to open the area followed shortly with the first test blast for riprap production on February 1, using 10 and 20 feet depth holes. Shot caprock from the initial blast was primarily used for building the dikes to contain the rock processing plant settling ponds and was also hauled to the plant for construction of a loading ramp. Some was stockpiled at TC2. After this, there was little activity in the Borrow Area other than dewatering until near the end of the construction period when three more shots to test riprap production were made in a single day on March 24 using 10 feet deep holes. The riprap test blasts were sampled for gradation testing.

The rock processing plant components included an impact crusher, a screening plant, and an auger type washer. It was tested and calibrated between February 1 and 2. Rock was processed from February 3 through March 4 when it was believed that enough of the filter and drain material had been produced. Disassembly of the plant began on March 7. This was completed and all components were either removed from the site or staged and awaiting transport from the site by March 16.

3.1.1 Test Cell 1

The stripping of peat and marl from TC1 began on January 21 and was complete on January 28. Excavation of the SCC began with the drilling of shot holes on February 3. The SCC was blasted in a series of three shots on February 8, 11, and 15. Work on the SCC excavation continued intermittently until completed on March 13.

Installation of the embankment open standpipe piezometers, in the foundation 20 feet downstream of the centerline, was completed on February 3 and 4, followed by embankment foundation preparation between February 10 and 16. Embankment construction began on February 16 and the embankment was essentially complete, except for minor grading on March 9.

Installation of the bench piezometer nests began on March 1 and continued until March 17. The upstream foundation piezometers and the core vibrating wire piezometers were installed on March 10 and 11. Installation of loggers in the open standpipe piezometers, flow meters in the fill and seepage recirculation piping, electronic and staff gage level indicators, and the RTU with logger was completed between March 16 and 30.

3.1.2 Test Cell 2

The stripping of peat and marl from TC2 began on January 17 and was complete on January 27. Excavation of the SCC began with the drilling of shot holes on February 19. The SCC was blasted in a series of five shots on January 21, 25, and 27 and February 1 and 4. Work on the SCC excavation continued intermittently until completed on March 23.

Installation of the embankment open standpipe piezometers, in the foundation 20 feet downstream of the centerline, was completed on February 2. Embankment foundation preparation started on January 31 and was completed on February 7. Embankment construction began on February 7. The rock trench for soil-bentonite cutoff wall construction was completed between February 11 and 13, and construction of the soil-bentonite cutoff wall was completed between February 21 and 28. The embankment was essentially complete, except for minor grading on March 24.

Installation of the bench piezometer nests began on March 16 and continued until March 28. The core vibrating wire piezometers were installed on March 28 and 29. Installation of loggers in the open standpipe piezometers, flow meters in the fill and seepage recirculation piping, electronic and staff gage level indicators, and the RTU with logger was completed between March 30 and April 9.

3.1.3 Construction Borings

During construction, a series of four geotechnical borings were completed in the bench between the embankment and SCC for each test cell. Each boring was located midway between the location of the inner and outer piezometer nests for the purpose of determining the piezometer screened zones. The drilling of these pilot holes was scheduled to allow it to proceed concurrently with early embankment construction, February 8 through 11 in TC2 and February 23 through 28 in TC1. Black & Veatch prepared boring logs of the pilot holes. The boring logs are contained in Appendix 8-17. The laboratory soil testing results completed on selected samples are contained in Table 3.

3.2 Test Cell Stripping

The surficial peat and marl was stripped from the entire footprint of each test cell including the SCC and bench between the embankments and SCCs. Stripping started with disking of the areas to promote drying. The majority of the stripping proceeded with towed agricultural scrapers and tractors. Areas with deeper or wet materials were completed with dozers pushing the soil into piles that were loaded to dump trucks with excavators. The stripped materials were transported to the perimeter of the active construction area and placed in berms as dikes to exclude water discharged from the SCC dewatering operations that was pumped to the areas outside. The peat perimeter berms acted to restrict the discharged water from flowing back onsite.

3.3 SCC Excavation

The SCC footprints were drilled and shot, generally with a pattern of three blast holes across the canal width. Typically the caprock was blasted for excavation by drilling blast holes through it into the underlying silty sand, and only the lower part of the holes was loaded. The length of the hole through the caprock was stemmed and contained no explosive. The result of the blast was primarily to lift the rock with only little of the explosive energy producing fragmentation.

The depths of the shot holes varied with time, generally being shortened to minimize mixing of the shot materials. The initial pattern used in TC2 consisted of a 20-foot central hole flanked by to 10-foot holes. This was modified to a 15-foot central hole in the last shot in TC2 and the first in TC1. The final two shots in TC1 used only 10-foot blast holes.

After being shot, the muck was excavated with hydraulic shovels (excavators) in three stages. First the caprock was removed and loaded to dump trucks hauling to the rock processing plant or other stockpiles. Then a narrow trench was excavated to full depth in the underlying soils. This was left open for a period of time to promote drainage of the material by the SCC dewatering operation prior to excavation to the final grade.

The soils were generally stockpiled directly along side the SCC by the excavators. Since the excavator reaches were much less than the width of the SCCs, this sometimes required rehandling of the material. An initial stockpile of the excavated soil would have to be picked up and cast further outside the SCC to allow room for further stockpiling of excavated soil.

3.4 SCC Dewatering

Two dewatering sumps were excavated in the SCC for each test cell. These were located in the southwest corner and along the north side of TC1 SCC and in the northwest and northeast corners of TC2 SCC. The first sump was excavated in each test cell immediately following and in the area of the first excavation blast. A second sump was then added in each SCC as excavation progressed. A mix of two types of pumps was used to discharge water from the sumps through the peat perimeter berms to outside of the work area. These included submersible, diesel hydraulic powered, axial flow pumps and diesel powered, centrifugal pumps. The types and location of the pumps for construction dewatering can be found in Appendix 8-19.

3.5 Embankment Foundation Treatment

Each test cell foundation was stripped of peat and marl as described above in Section 3.2. Further preparation of the foundations was provided only under the embankment cores. Loose caprock was removed with hand tools and a small excavator. The core area of each cell was then swept with a circular power broom mounted on a small loader before select fill placement. Also after cleaning, the area was maintained moist by periodic sprinkling passes with a water truck until fill placement. The specification required the application of slush grout prior to select fill placement, but this was deleted because the fill bonded well to the foundation without it.

3.6 Embankment Fill Placement

Embankment rockfill, select fill, random fill, and blanket filters and drain were placed by dump truck, spread by dozer, and then appropriately compacted with passes of a smooth drum vibratory roller. Lift thicknesses and initial compaction efforts are given in the specifications. Chimney and finger, filter and drain materials were placed with the bucket of a front end loader between successive lifts of the adjacent embankment zones and spread with hand tools, shovels and rakes. Survey control was provided by Barnard Construction.

Essentially all embankment fill was obtained from the test cell SCC excavation. As noted above in Section 3.3 the caprock was mined first from the SCCs. This was mostly hauled to the rock processing plant for crushing and screening to filter and drain materials, some was stockpiled for use as rockfill and riprap. The soils below the caprock were stockpiled adjacent to the SCCs for use as random and select fill. A small amount of shot rock from the Borrow Area was hauled to TC2 for embankment construction with the majority of the material coming from the SCCs.

Laboratory tests completed on the embankment select and random fills included standard Proctor, nuclear density, sand cone density, and moisture content. The testing and locations were assigned by field inspection personnel. Moisture content was tested periodically on selected stockpiles to measure the rate at which they drained. The embankment construction and stockpile testing results are summarized in Appendix 8-21.

As a test of the rate at which it could dry, select fill was spread into twelve small stockpiles about 2 to 3 feet deep on the northeast bench of TC1, “the select fill farm.” The moisture was tested daily February 15 through 19. The data for the piles was arranged in order of descending initial moisture content and plotted as continuous series to track the moisture change trend. This is presented in Figure 2 as “relative days.” The stockpiles are much smaller than those typically used for the excavated materials at the test cell site. The graph shows a drying trend which is probably enhanced by the small size of the piles producing better exposure of the material to evaporation.

3.7 Test Cell 2 Soil-Bentonite Cutoff Wall Construction

A rock trencher was used to cut an initial trench through TC2 foundation caprock. This trench was backfilled with the trench spoil to allow embankment core select fill placement to the level for a working platform for cutoff wall construction. The remainder of the trench was then excavated with a long-arm excavator using bentonite slurry for trench support and a native soil and bentonite slurry backfill. The excavated spoil was mixed with bentonite using a dozer to work and blend the material and blade the blended slurry material back into the trench. These operations proceeded in sequence around the perimeter of the soil-bentonite cutoff wall until complete.

Testing completed on the backfill slurry during construction included slump cone, sand content, and density by mud balance. Samples of the slurry were sent for laboratory testing including specific gravity, gradation, and hydraulic conductivity. The results of the laboratory testing are given in Table 4.

3.8 Construction Equipment

Major equipment used for test cell construction included hydraulic excavators, articulated dump trucks, wheel loaders, dozers, smooth drum vibratory rollers, and drills in addition to the rock processing equipment. The majority of the rolling equipment was manufactured by Caterpillar, Inc. Table 5 summarizes the major equipment used by Barnard Construction and the various subcontractors. In addition to this, ancillary equipment included pumps for dewatering and slurry delivery, generators, light plants, and tanks for fuel storage.

3.9 Riprap Test Blasts

Four test blasts for riprap (erosion protection stone) were completed in the Borrow Area. The spacing and burden of the blast holes was varied to test the affect on the product gradation. All the patterns were square: 20 feet by 20 feet, 25 feet by 25 feet, 28 feet by 28 feet, and 35 feet by 35 feet. The shot rock produced by each blast was sampled and tested for gradation. The product sample gradations are given in Table 6 and shown on Figure 3.

3.10 Equipment Production

Construction equipment productivity was tracked on the day shift, during the weekdays, between February 22 and March 22. The daily construction equipment logs are included as Appendix 8-19. The hours of production are summarized in Tables 7 and 8 for Test Cells 1 and 2 respectively. Similarly the production (yards excavated or placed over a period of time) was tracked on selected pieces of equipment. The excavation and fill logs can also be found in Appendix 8-19.

Construction efficiency on the test cells was affected by the small work area and the tight construction schedule imposed. Because of the very small scale of the test cell construction, the construction operations were confined to small areas with resulting inefficiencies. Equipment hauling for fill placement was often waiting for a previous placement to be spread and compacted. This of course also affected the efficiency of excavation equipment that was then spending significant time waiting for the haulers to return.

The production efficiency of the test cell construction program was unbalanced by the small areas of fill placement. Fill placement rate determined the excavation and hauling rates. Production efficiency was also lowered by the need to stockpile some materials for drying, sometimes even re-stockpiling.

3.11 Construction Rock Quality Testing

During the construction period, the caprock core remaining from the December geotechnical borings at the test cell site was sent for rock quality testing along with samples of the filter and drain produced by the rock processing plant to a geotechnical laboratory for the following testing:

- C88, Standard Test Method for the Soundness of Aggregates by the Use of Sodium or Magnesium Sulfate.
- ASTM C131, Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.
- ASTM C535, Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.

The testing results are contained in Table 9.

3.12 Test Cell Breaching

One of the permitting requirements was that the Test Cells should be breached on conclusion of the tests to ensure that they could not permanently retain water. This operation was carried out by Barnard between June 14 and 15, 2005 using a CAT 330C tracked excavator. Embankment material was cast to the side on the embankment to block the crest road and prevent vehicles driving into the excavation.

Groundwater level was at approximately 8 ft elevation due to persistent heavy rain over a prolonged period. This meant that the fill to caprock interface was covered by water and could not be observed.

The excavation process was observed by Black & Veatch and fill density tests were carried out by Nodarse & Associates.

3.12.1 Test Cell 1

The North side of Test Cell 1 was breached on June 14, 2005.

The riprap and bedding were in good condition when removed; with limited fetch, waves were small throughout the life of the embankment so no damage was expected.

During construction, the select fill core was placed in a wet condition and compaction was limited to construction traffic and the weight of material above. When excavated, the select fill appeared competent and had hardened or consolidated. Moisture content and density tests were carried out and the results are shown in Table 10 below. In general terms moisture content was lower and density higher towards the base of the embankment. This is consistent with consolidation due to the pressure of fill above.

The inclined filter and drain system was exposed downstream of the core. The contact between the core and the filter was carefully examined for signs of fines migration. No signs were found. In fact, there were no signs of water movement in the filter; the slightly dirty wet residue remaining from the original crushing operation still coated the rock particles.

The downstream rockfill section was competent and did not release water when excavated. This is consistent with the observation that the select fill core was watertight.

The general water level in the area was approximately 8 feet elevation and the interface between the fill and caprock was submerged.

3.12.2 Test Cell 2

The South side of Test Cell 2 was breached starting June 14, 2005 and it was completed on June 15, 2005.

The upstream rockfill was saturated when the test cell was full and it was still wet in places; free water drained as excavation progressed.

The interface between the random fill and central select fill zone was difficult to distinguish because the materials are the same apart from particle size. Both the select and random fill materials appeared competent near the base of the embankment but softened at higher elevations. This was evident from the movement of the material underfoot ('pumping') at higher elevations. Moisture content and density tests also support this observation as shown in Table 10 below.

The inclined filter and drain system was exposed downstream of the core. The contact between the core and the filter was carefully examined for signs of fines migration. No signs were found. In fact, there were no signs of water movement in the filter; the slightly dirty wet residue remaining from the original crushing operation still coated the rock particles.

The general water level in the area was approximately 8 feet elevation and the interface between the fill and caprock was submerged.

4. TEST CELL SEEPAGE TESTING

After the test cell embankments were constructed, the test cells were filled with water from the Primary Canal and the associated SCCs. Field data was collected regarding the filling rates and the water elevations of the test cells and the SCCs. This section describes the field data collected, the testing completed, and the results of the filling and drawdown of both test cells TC1 and TC2, and the drawdown of the SCC in TC1 during this field testing and monitoring.

4.1 Background Conditions

The three sets of three background piezometers with depths of 25, 60, and 100 feet below the ground surface were located approximately 500 feet south of the west embankment of TC1, approximately 500 feet north of the east embankment of TC2, and midway between TC1 and TC2 approximately 500 feet off the embankments of both TC1 and TC2. The water levels recorded by the miniTroll data loggers in each piezometer were periodically downloaded into a handheld data logger and the results plotted. Additionally, two additional 25 foot depth piezometers were constructed approximately 500 feet south of the east and west embankments of TC2. The water levels in these two additional piezometers were periodically measured with a water level indicator.

This layout and distance of background groundwater instrumentation in relation to the test cells was designed to record regional groundwater levels at a distance far enough away from the test cells to be minimally affected by filling and dewatering. However, during the field testing and monitoring period it was discovered that the background groundwater levels were influenced by the level of the Primary Canal and local rainfall events. Rainfall events that were in the area but not local to the site also influenced the site indirectly through the controlled levels of the Primary Canal.

A staff gauge was installed in the primary canal on March 22. From that time until the end of the monitoring period during the week of May 16 the water elevation in the primary canal ranged from approximately 4 to 7.8 feet. The levels in the Primary Canal are controlled by the local sugar producer through pumping, and this pumping is influenced by rainfall events. A review of the data during this time period suggests that the background water level elevation was in the range of 6.2 to 7.0 feet.

4.2 TC1

TC1 was filled in three phases, to elevations 12 (about 4 feet depth) and 16 feet (about 8 feet depth) and finally to an elevation of 20 feet (about 12 feet depth) where the water level was held for the remainder of the monitoring period. A final phase of testing included the lowering of the TC1 SCC to elevation 2.

4.2.1 Filling

The filling of TC1 started on April 4 with water from the Primary Canal. The Primary Canal level (as indicated by the staff gage) was initially at elevation 6.35 feet and the TC1 SCC level was at elevation 5.95. Pumping began from the TC1 SCC the following day when the level on the TC1 SCC staff gage was at elevation 6.05.

TC1 filling attained the 4 foot water depth (12-foot water elevation) on the afternoon of April 5. The filling proceeded to approximately elevation 15.5 feet when it was decided, because of wet conditions developing on the bench area between the embankment and the canal, to stop filling and hold water level steady for a period of observation. The test cell water level was held at elevation 14 for approximately 13 days from April 7 to April 20. Based on an assessment of the seepage losses from TC1 up to the holding elevation, it was calculated that the originally envisaged pumping capacity would be insufficient to fill the cell to elevation 20. During the hold period an additional 12" pump was mobilized to provide the necessary capacity to take the test to full depth.

The filling then continued to the target water level elevation of 20 which was achieved on April 21. The test cell water level was held at near elevation 20 feet following this, by combined pumping from both the Primary Canal and SCC for the remainder of the monitoring, except for one brief period at the end of the test as noted below in Section 4.2.2.

During the filling of TC1, the appearance of water on the bench between the embankment and the SCC was almost immediately noted. Springs or boils developed at identifiable cracks and solution holes along with general areas of seepage. With continued filling, new springs continued to develop and the existing ones apparently increased in discharge. The east and north benches were the wettest, but springs and seeps were common on all sides and most numerous nearer to the embankment. The filling was slowed because of the seepage, and the springs were marked and observed. All the spring discharges appeared clear, however, and filling was completed to the target elevation.

4.2.2 Canal Drawdown

The drawdown of TC1 SCC for the final testing phase began on May 5 from a water elevation of 7.51 feet. The water level elevation in the test cell was at 18.18 feet because pumping from the SCC had been temporarily suspended on the previous day due to heavy rain leading to test cell levels above the target elevation. The test cell level was returned to approximately elevation 20 feet, and the water level in the canal was lowered at a steady rate until it reached the target elevation of 2.0 feet on May 10 using the 12-inch pump moved from the Primary Canal to the SCC southwest corner for that purpose.

4.2.3 Cell Drawdown

The TC1 drawdown began on May 17 at 5:00 pm from the water level elevation of approximately 20 feet. At that time the elevation of the SCC was 1.99 feet.

4.3 TC2

TC2 was filled in two phases, to an initial elevation of 14 (about 6 feet depth) and then advanced to the target water level elevation of 20 feet (about 12 feet depth).

4.3.1 Filling

The filling of TC2 started on April 14 with water from the Primary Canal. The Primary Canal staff gauge was at elevation 6.60 feet and the TC2 SCC staff gauge was at 6.47 elevation feet. Water was added from the TC2 SCC soon after the start of filling.

The filling in TC2 attained the 6 foot water depth (14 foot water elevation) on April 15. The holding period at this water elevation lasted for a period of approximately 4 days from April 15 to April 18. The filling continued to the target water level elevation of 20 which was achieved on April 19.

4.3.2 Cell Drawdown

The TC2 drawdown began on May 16 at approximately 11:00 AM from the water level elevation of approximately 20 feet. At that time the elevation in the SCC was 6.70 feet.

4.4 Modifications to Makeup and Seepage Recirculation System

During the initial part of the testing and monitoring, the 8-inch pump installed in TC1 SCC did not have a high enough capacity to recirculate the seepage into the SCC. Also, the maximum range on the installed flow meters was often too low to register the large flow required for initial test cell filling and for seepage recirculation. A second, 12-inch, seepage recirculation pump and piping were installed in TC1 and two portable ultrasonic flow meters were obtained to be used when the installed flow meter range maximum was exceeded.

Once both test cells were full, the 8-inch pump at the Primary Canal was of sufficient capacity to provide makeup water for maintenance of test cell water levels. The 12-inch pump was then moved to southwest corner of TC1 SCC to discharge water from the SCC to outside the peat perimeter berm around the test cell area. The pump was then used for the drawdown of TC1 SCC.

5. CONSTRUCTION PROGAM ISSUES AND LESSONS LEARNED

Among the objectives of the Test Cell Program was to obtain construction information and experience with the site and the site materials to guide the design of and planning for the larger EAA Reservoir A-1. In this regard, valuable experience and data were garnered during the construction program.

5.1 Site Condition

The Test Cell Program construction site and that for the EAA Reservoir A-1 have little relief, are poorly drained, and are covered with a fine grained, organic soil. The groundwater table is just below the ground surface. As such, control of surface water and water discharges is an essential issue.

The surficial peat overlying the caprock has low bearing capacity when dry and even less when wet. The peat produces a very dusty environment when dry and exposed. It is slippery when wet. The marl becomes soft when wet. It pumps and ruts. Rubber tired equipment and vehicles traversing on these soils can easily become mired and stuck. The peat and marl are not suitable road subgrade and must be removed from beneath haul and access roads. Frequently, this stripping lowers the ground surface to below the water table. Any roads must be constructed with an adequately thick and coarse grained base to prevent capillary action from saturating it from the shallow groundwater.

Disposal of excavation dewatering discharge is a concern because of the flat site. At the test cell site, a berm was built around the site primarily with the stripped surficial soils. The excavation dewatering discharge was then piped beyond this berm. The berm, however did not adequately exclude the discharged water in areas of high pumping volume such as northeast of the site. In that area the discharged water seeped back under the berm causing a problem of standing water in work areas such as the rock processing plant. The location of disposal for use in construction of the larger EAA Reservoir A-1 should consider potential impacts on bordering agricultural fields and other properties that could be flooded.

5.2 Foundation Conditions

The caprock is planned as the bearing surface for the primary EAA Reservoir A-1 containment structure. The caprock thickness varies between 0 and 14 feet over the EAA Reservoir A-1 site. The condition of the caprock also varies greatly. It contains channels and porous zones from solution by fresh water. The dissolution is most advanced near the top but penetrates the entire thickness. Prediction of the extent of different zones of weathering/caprock condition is difficult and must be viewed as a risk item. At the test cell site, surface conditions varied over distances of tens of feet and were not predictable until the surface was brushed. Isolated boreholes do not provide a reliable method of assessing caprock condition over large areas and we understand that previous geophysical investigations, which can give continuous information about ground conditions, have proved unsuccessful in the Everglades.

The selection of the type of containment structure to be used must consider the wide variability of the foundation conditions. Where the caprock is adequately thick it would support large rigid structures, but over the thinner areas, settlements could be induced. Differential settlement could be a detriment to the performance of rigid structures due to the potential for disjointing and leakage.

The necessary foundation treatment for structures of different design must also be considered. Treatments will be necessary to control seepage along the interface between the structure and the caprock and in the more permeable layers near the top of the caprock. These treatments could include cleaning, overexcavation of cavernous material, grouting, and installation of a cutoff wall. However the design should limit the necessary excavation of the caprock for foundation improvement. Excavation further thins the caprock and increases the likelihood of differential settlements. It also tends to damage and loosen the rock below.

5.3 Construction Materials

5.3.1 General

Potential construction materials available near surface include the limestone caprock and underlying silty carbonate sand of the Fort Thompson Formation.

The material of the underlying Caloosahatchee Formation includes primarily sand and gravel that could be screened and blended to produce filter, drain, and concrete aggregate or used directly as random fill but it is at depths below those expected for project excavations. The Caloosahatchee Formation also contains little limestone.

The caprock of the Fort Thompson Formation is the only rock available on a usable scale as rockfill, riprap, or for the production crushed stone for filter, drain, and concrete aggregate. While other hard limestone lenses exist below this in the Fort Thompson Formation, they are too thin and too deep to allow economical mining and separating for construction materials.

5.3.2 Select and Random Fill

The carbonate sands of the Fort Thompson Formation which underlying the caprock contain an average of 22 percent silt. They therefore have a relatively low permeability (laboratory tests on samples compacted to around 90% ranged from 1.3×10^{-6} centimeters per second to 8.8×10^{-4} centimeters per second, most were about 3×10^{-5} centimeters per second) and are suitable as embankment core material. They are also suitable as random fill.

The carbonate sand of the Fort Thompson Formation used as select and random fill in construction of the test cells is saturated in situ and drains very slowly when excavated and stockpiled. Material excavated in the saturated condition could have the properties of thick slurry. Efforts made to control the moisture content of the material during test cell construction included:

- Excavation and dewatering of a deep drainage trench ahead of bulk excavation in the SCCs to promote drainage.
- Stockpiling.
- Spreading and restockpiling.

Despite these efforts, the moisture content of material placed in the random and select fills was high, above optimum for compaction, even for material that had been stockpiled for several weeks. In fill areas, this material was rutted deeply by equipment and it pumped and adhered to the smooth drum compactors. As a result, in place densities did not meet the expectations of the specifications.

The high moisture contents also lead to unreliable sand cone density determinations. The wet fill tended to squeeze into the test holes. As a result, nuclear density testing was primarily used on the fills.

Triaxial tests have been completed on remolded samples of select fill. The results are shown in Appendix 8-24.

Another property of the carbonate sands is that they harden on drying. They attain a considerable strength that makes excavation with hand tools very difficult. The strength potential that they attain is currently being evaluated in addition to the possibility of shrinkage that could cause cracking.

Despite the fill moisture control concern, the test cell embankments performed well with no piping or slope failures observed. The density tests and moisture contents of the select fill at the time of breaching are given in Table 10 below.

5.3.3 Filter, Drain, and Concrete Aggregate Materials

The results of rock quality testing on the filter and drain produced for the test cell construction is contained in Table 9. The test results indicate that a high quality, durable crushed aggregate can be produced from the caprock. However, it must be stressed that any the testing completed to date on caprock cores and any material processed from the caprock is representative of the upper limit of the caprock quality.

Much of the in situ material is of a low quality that would not be suitable for processing to filter, drain, and concrete aggregate and it is destroyed during the coring process. The average core recoveries in both the test cell program borings and the previous investigations were about 50%. Some of the core recovered is also soft and porous. At present, all indications are that much of the caprock mined would not be suitable for filter, drain, and aggregate production.

The caprock is also associated with and contains peat and marl in the solution cavities. When it is blasted with the underlying silty sand, it becomes further contaminated with the silty sand. It produces an unusually dirty crusher feed. There would be unusually large amounts of waste generated by any rock mining and processing.

5.3.4 Rockfill

The Test Cell Program specifications required no more than 10 percent by weight of rockfill to pass the 1-inch screen. Visually, much of it appeared to contain more than this. Locally the rockfill would rut under passing equipment.

This is again a consequence of the underlying silty sand being mixed with the caprock when it is blasted. Also when the caprock was excavated, the cleanest, upper portions from the shots were directed to the rock processing plant for crushing. The rockfill was taken from deeper more contaminated portion of the shot material. The rockfill zones of the test cell embankments performed well despite the entrained fines.

5.3.5 Riprap (Erosion Protection Stone)

The product gradation from the four riprap test blasts at the test cell Borrow Area is given on Table 6 and graphed on Figure 3. Despite the wide range of shot hole spacing (20 by 20 feet to 35 by 35 feet), the gradations are not very different. In the 3-feet plus gradation size the largest stones produced weighed 1,088 and 1,171 pounds from the 25 by 25 feet and 35 by 35 feet test blast patterns respectively. The average weight of the 3-foot plus stones was 512, 596, 605, and 721 pounds for the respective test blast patterns, from closest to widest spacing. The small variation in gradation between the blast patterns and the small range in the size of the largest stones suggests that the product gradation is influenced more by the intrinsic discontinuities in the rock than the shot pattern. The gradation is a function of the bedding thickness, joint spacing, and the distribution of solution channels.

The largest size of the stone produced in these test blasts is as coarse as can be produced from the material at the site and probably is about 1,100 pounds. The largest blocks of riprap produced tend to be slabby due to the horizontal bedding. Also, because of porosity, vugs, and solution channels, the caprock is lighter than a typical solid limestone. These properties should be accounted for in plans to use the material for erosion protection.

5.4 Available Quantities

At the Test Cell Program site, more than sufficient quantities of the materials were available from the SCC excavations for the embankment construction. The SCC excavations were about 20 feet deep and about twice the length of the embankments. This ratio will be closer to 1:1 for the EAA Reservoir A-1, so it is likely that borrow areas will needed to augment supplies of construction materials available from the planned excavations. The amount necessary will depend on the embankment and canal sections and alignments adopted. Conversely, the most economical design would result from minimizing the final borrow area volume and excess excavation of materials.

Supplies of caprock will probably be the most critical concern. The caprock is uniformly almost 5 feet thick over the test cell site, but over the entire EAA Reservoir A-1 site the caprock thickness ranges from 0 to over 14 feet as determined from the 2003 and early 2004 borings. The average thickness from those borings is about 5.7 feet. Embankment designs with large rockfill, riprap, or crushed rock zones or designs using large amounts of concrete would require the opening of borrow areas to supply the rock.

6. TESTING AND MONITORING RESULTS

The test cell pumping and level data from the testing and monitoring phase is contained in Appendix 8-22. The field testing and monitoring progressed through three phases:

1. Staged filling and stabilization of each test cell water level at the target elevation of 20 feet.
2. An attempt to stabilize the SCC water levels with the general groundwater potentiometric surface (background water level) while maintaining the test cell water levels at the target elevation.
3. Drawdown of TC1 SCC water level to below the caprock.

The purpose of the first phase was to establish the basic seepage rates from the two test cells and determine the relative effect of the soil-bentonite cutoff wall on seepage. The second phase attempted to determine the relative proportions of the test cell seepage that was intercepted by the SCCs and that portion of the seepage going to the area outside the test cell site, or background. The third phase was conducted in TC1 only to observe and evaluate seepage through the caprock on the SCC banks.

6.1.1 Test Cell Stabilization at the Target Elevation

When the test cell water levels were stabilized at around the target elevation of 20 feet, filling from the Primary Canal was discontinued and the test cell water levels were maintained for a time by pumping with the SCC seepage recirculation pumps only. TC1 water level was stabilized at essentially elevation 20 feet between 4:00 PM on April 22 and 7:00 PM on April 23 with a pumping rate of 3,900 gallons per minute (gpm). In TC2 the water level drifted up slowly as the pumping rate was slowly lowered. Then at about 5:00 PM on April 23 as the pumping rate was lowered below 1,900 gpm, TC2 water level slowly began to drift down.

The seepage rate in TC1 (no soil-bentonite cutoff wall) of 3900 gpm, was almost twice that in TC2 (with soil-bentonite cutoff wall) at 1,900 gpm, with the test cell water levels stabilized around elevation 20 feet. Differences in the test cell geometry have not been accounted for here.

6.1.2 Stabilization of the Test Cell SCCs with Background

During the next phase of testing and monitoring in the test cells, the SCC recirculation pumping rates were lowered to allow the SCC water levels to rise in response to increasing background water levels. Makeup water was provided from the Primary Canal to maintain the test cell levels at the target elevation of 20 feet. With the SCC level very near the background water level, very little water should be coming into the SCC from outside (background). In this situation, the water pumping rate from the SCC into the test cell should be nearly equal to the seepage from the test cell into the SCC, and the makeup water pumping rate from the Primary Canal should be very nearly equal to the seepage rate bypassing the SCC and entering the background

The practical problem that was encountered during this phase was that the background water level and the background piezometer readings vary significantly with time and location. A unique background level could not be established. Under these circumstances, it was not possible to accurately determine the proportions of test cell seepages going into the SCCs and that bypassing them. However, it does appear that the largest portion of the seepage from the test cells was intercepted by the SCC.

This phase of the testing and monitoring continued from the morning of April 3 to the afternoon of April 12. For a short period the makeup pumping to both test cells was increased to raise the water levels. If one excludes the data from that period, so that only the pumping data from those periods of time when the test cell water levels were most stable are included, the pumping from the Primary Canal is only a small percentage of the total pumping to the test cells. The average pumping rate from the Primary Canal to TC1 was 510 gpm (about 12 percent of the total) compared to a 3670 gpm average pumping rate from TC1 SCC. The average pumping rate from the Primary Canal to TC2 was 259 gpm (about 14 percent of the total) compared to a 1547 gpm average pumping rate from TC2 SCC.

6.1.3 TC1 SCC Drawdown

Following the second phase of testing and monitoring, the water level in TC1 SCC was drawn down to about elevation 2 feet using the 12-inch pump moved from the Primary Canal to the southwest corner of TC1 SCC for this purpose. As the SCC water level was drawdown, the recirculation pumping rate from the SCC to the test cell had to be increased to maintain the test cell level at the target elevation. When the SCC water level was drawn down about half way, the 12-inch SCC recirculation pump no longer had sufficient capacity to maintain the test cell water level, and the 8-inch pump was started to augment the 12-inch pump supply.

During the drawdown some slope instability, in the form of sloughs in the silty sand, was noted in the SCC slopes. Also a few plumes of turbid water were noted. However, once the full drawdown was attained and stabilized, these signs of instability and deterioration ceased to be seen. During construction, Barnard placed soil against the vertical face of the exposed caprock in the SCC to produce a uniform slope, so generally water could not be observed coming directly from the caprock. However, the seepage through the caprock appeared to be greatest to the north and east side, and the majority did appear to issue from near the base of caprock.

With the water levels in the SCC and test cell stabilized with the pumping during this phase of testing, the pumping into the test cell theoretically would equal the test cell seepage, and the pumping from the SCC to the outside would equal the background seepage into the SCC. The water level in TC1 SCC was lowered constantly from elevation 7.54 feet at 6:00 AM on May 4 to 1.81 feet at 11:00 AM on May 11. The water level in test cell was closest to stable (drifting upward very slowly) after the morning of May 9 when the pumping rate into the test cell averaged 4,743 gpm. It appears that the seepage from the test cell with the SCC drawn down was about 4,700 gpm, and the seepage from background into the SCC was somewhat less than 4,800 gpm.

7. CONCLUSIONS AND RECOMMENDATIONS

The Test Cell Program demonstrated that:

- With proper design, stable embankments can be constructed from the locally available earth materials using proper handling and construction techniques
- The silty sands below the caprock have higher moisture content than is required for good compaction and drain very slowly when stockpiled
- A constructed cutoff wall will reduce seepage from the EAA Reservoir A-1. The make-up water rate to the test cell with the soil bentonite cutoff wall was about half of that to the cell without a soil-bentonite cutoff wall
- The caprock is a conduit for near surface seepage, though the relative amount has not been established
- The total depth, and stratigraphy at depth, of the surficial aquifer system at the EAA Reservoir A-1 are currently not well defined
- Crushed stone construction materials such as filter, drain, and concrete aggregate can be produced from the local caprock but the production efficiency will be limited by the variable properties of the caprock and entrainment of the underlying silty sands during mining
- Riprap can be produced from the local caprock. However, it is of limited size and whether or not the size is adequate for the EAA Reservoir A-1 has not been determined

The embankments constructed in both test cells were stable and watertight. The major concern to construction was the relatively high moisture content in the silty, carbonate sands of the Fort Thompson Formation that were used for select and random fill. Efforts to drain the material were not sufficient to bring it to the proper moisture content for compaction. Despite this the embankments were stable.

The soil-bentonite cutoff wall installed in TC2 was clearly effective in reducing foundation seepage relative to that in TC1. It was apparent from the wet surface condition around TC1 that a significant amount of the foundation seepage was passing through the caprock, but the relative amounts of caprock versus deeper seepage cannot be evaluated without further analysis and modeling.

Further analysis and modeling will also be necessary to evaluate the appropriate depth for a soil-bentonite cutoff wall. Various publications in the past have given differing interpretations of the properties and the depth of the permeable strata (the surficial aquifer system) underlying the EAA Reservoir. Various models for assessing reservoir seepage based on these interpretations have been developed. Few of the explorations completed to date have penetrated beyond a depth of 50 feet. Deeper borings using down-hole geophysics to delimit the various strata would be useful to produce a definitive model for seepage analysis and estimation.

The caprock is a source of construction stone. It can be used for riprap, rockfill, and crushed stone products. It does contain soft, porous zones though. Much of the material mined will break down during processing, producing a large percentage of waste and increasing the amount of washing needed during processing for filter, drain, and concrete aggregate. A two stage crushing operation could also be helpful.

When shot for excavation, the underlying silty, carbonate sand tends to be entrained into the caprock increasing the fines content of the rockfill and crusher feed. The depth of excavation into a shot can be limited to minimize this entrainment. This will require a larger area of borrow to provide an adequate volume.

Riprap test blasts produced large size stone suitable for riprap but the maximum size gradation that can be produced will be limited by the natural discontinuities in the caprock, with the weight of the largest stones being about 1,100 pounds. The larger stones also tend to be tabular in shape and contain solution cavities and channels.

The experiences during the Test Cell Program support the following recommendations:

- The EAA Reservoir A-1 containment structure should include a cutoff wall into the foundation. The final depth of this cutoff wall will depend on future investigations and modeling
- Further geotechnical borings should be conducted to define the deeper stratigraphy around the EAA Reservoir A-1 for seepage modeling
- Testing should be conducted to determine the rate at which the silty, carbonate sand below the caprock drains when stockpiled and to investigate more efficient methods of drying
- The construction scheduling for the EAA Reservoir A-1 should consider time allotted for draining and drying of the silty, carbonate sand prior to use in embankment construction
- The depth of excavation into a shot for rockfill or crusher feed should be limited to limit the inclusion of silty, carbonate sand entrained in the lower part

- A two stage crushing operation for the production of filter material should be investigated

Currently a number of 100-foot deep geotechnical borings are planned around the periphery of the EAA Reservoir A-1. At least two of these boring should be extended to a depth of 250 feet to clearly define the base of the surficial aquifer system. This is needed to determine the depth limits for the models used to analyze reservoir seepage. Downhole geophysical logging should also be added to the program to further define the limit of the individual strata and the physical properties.

During construction of the test cells, efforts to lower the moisture in the silty, carbonate sands used for select and random fill were inadequate to produce moisture contents near optimum. The material is saturated in situ and drains very slowly. Efforts to drain it included excavation of an initial trench for drainage prior to bulk excavation and stockpiling above ground.

During construction of the EAA Reservoir A-1, more time should be allotted for borrow moisture control. The initial drainage trenches should be opened earlier and more time allowed for the material to drain before bulk excavation. The material will need to be stockpiled for longer periods of time and the stockpiles turned to expose the material for evaporation to speed the drying. Testing such as periodic testing and manipulation of an existing stockpile could be used determine the time that will be needed.

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TABLES

Table 1 December 2004 Boring Rock Testing Results

Boring	% Recovery	% RQD	UCS
TC-01	54	46	3970
TC-01			4740
TC-02	42	21	
TC-03	34	0	
TC-04	48	28	4370
TC-04			6060
TC-05	72	40	8160
TC-05			7310
TC-06	19	10	
TC-07	25	20	
TC-08	44	26	7310
TC-08			5700
TC-09	56	50	6650
7C-09			4080
TC-10	16	14	
BA-01	50	22	10460
BA-02	48	42	3110
BA-02			8590
BA-03	50	28	5970
BA-04	53	31	
BA-05	56	42	
BA-06	35	22	3450
BA-06			3170
BA-07	32	6	
BA-08	66	52	10350
BA-08			3310
BA-09	70	52	
BA-10	48	30	7110
BA-10			10350

Table 2 December 2004 Boring Soil Testing Results

				Gradation: Percent Passing Sieve										Hydrometer	
Boring	**Depth**	**Moisture Content %**	**Carbonate Content %**	**3/4-- inch**	**3/8- inch**	**#4**	**#10**	**#20**	**#40**	**#60**	**#100**	**#140**	**#200**	**Silt**	**Clay**
BA-01	5.5 - 7		87.4												
BA-01	28.5 - 30		29.1												
BA-01	43.5 - 45		10.9												
BA-01	48.5 - 50			100	97.9	97.6	97.4	95.5	89.4	80.6	58.1	17.5	7.5		
BA-02	18.5 - 20			81.3	65.6	55.4	47.1	37.3	28.7	20.9	15.7	12.9	11.4		
BA-03	8.5 - 10		82.6												
BA-03	13.5 - 15			98.6	97.2	93.1	85.6	74.3	58.3	45.2	36.9	31	28.1		
BA-03	29 - 30.5		40.5												
BA-03	43.5 - 45		37.3												
BA-03	48.5 - 50			100	100	99.3	96.5		88.9	86.1	73.7		21.9	18.3	3.6
BA-04	6 - 7.5		87.7												
BA-04	13.5 - 15			70.5	63.6	60.6	55.4		43.4		30	20.1	20.2	14.4	5.8
BA-04	18.5 - 20		82.7												
BA-04	33.5 - 35		31.4												
BA-04	48.5 - 50			100	100	99.3	97.7	94.5	91.3	88.3	50.1	14.5	6.4		
BA-05	8.5 - 10		86.2												
BA-05	28.5 - 30		36.8	100	93.4	79.2	63.5	54.6	50.2	45.7	31.1	17.2	11.4		
BA-05	38.5 - 40		31.4												
BA-06	6 - 7.5		84.7												
BA-06	13.5 - 15			100	100	99.1	95.4	87.9	77.6	65.7	54.4	45.5	39.5		
BA-06	28.5 - 30		27.6												
BA-06	48.5 - 50		34.6												
BA-07	6 - 7.5		83												
BA-07	18.5 - 20		84.8												
BA-07	38.5 - 40		28.5												

				Gradation: Percent Passing Sieve										Hydrometer	
Boring	Depth	Moisture Content %	Carbonate Content %	3/4-- inch	3/8- inch	#4	#10	#20	#40	#60	#100	#140	#200	Silt	Clay
BA-07	43.5 -45			100	98.5	89.6	78.9	70.8	66.7	62.4	36.3	13.8	8.4		
BA-08	8.5 - 10		85.9												
BA-08	13.5 - 15		83.7												
BA-08	38.5 - 40		32												
BA-08	28.5 - 30			93.9	83	77	70.1	63.6	59.1	53.2	32.9	16.9	12.2		
BA-09	13.5 - 15			100	96.7	91.3	84.9	76.9	67.6	54.8	43.1	34.1	29		
BA-09	23.5 - 25		78.5												
BA-09	38.5 - 40		25.3												
BA-09	43.5 - 45			100	97.8	95	88.5		82.6	81	72.6		39.8	33.2	6.6
BA-10	13.5 - 15		83.5												
BA-10	33.5 - 35		26.7												
TC-01	38.5 - 40	24.5											10.3		
TC-01	48.5 - 50			91.1	97	81.9	72		56.1	48.1	34.1		10.4	8.5	1.9
TC-02	8.3 - 9.8			100	93.6	88.7	82	66.4	46.5	32.8	26.7	24	22.1		
TC-03	13.5 - 15			100	98.2	95.2	90.8	84.1	74	60.6	45.1	32.6	26.6		
TC-04	6.5 - 9			100	88.4	7302	58.6		38.9	32.6	27.2		22.1	13.3	8.8
TC-04	48.5 - 50	26.8											6.7		
TC-05	48.5 - 50			87.4	86.8	84.3	79.9	73.4	69.3	66.3	53.7	22.3	13.2		
TC-06	8 - 9.5	17.2											11.5		
TC-06	18.5 - 20			100	97.8	91.8	79.8	61.2	45.4	31.5	25.4	17.4	13.1		
TC-06	28.5 - 30			100	93.2	85.9	78.1		68.7	64.2	39.9		9.2	7.8	1.4
TC-07	28.5 - 30			100	75.5	65	56.2	51.5	49	46.5	35.8	16.6	9.5		
TC-08	43.5 - 45			100	100	98.2	96	95.4	95.2	94.7	85.8	45.4	28		
TC-09	23.5 - 25	26.2											23.3		
TC-10	18.5 - 20			100	100	97	93.4	86	79.2	72.3	65.8	49.6	44.9		

Table 3 Construction Boring Soil Testing Results

				Gradation: Percent Passing Sieve											
Boring	**Depth**	**Moisture Content %**	**Carbonate Content %**	**2-inch**	**1-inch**	**3/4-inch**	**3/8-inch**	**#4**	**#10**	**#20**	**#40**	**#60**	**#100**	**#140**	**#200**
TC1-N	5.5 - 7	15				100	94.9	83.2	70.8	56.8	44.4	34.3	27.7	23.9	21.5
TC1-N	8.5 - 10		79												
TC1-N	33.5 - 35	26				100	98.3	98.1	95.6	90.5	85	72.5	30.9	9	5.6
TC1-N	53.5 - 55	15		100	75	71.2	67.8	61.6	53.2	43.1	34.8	27.2	19.9	15.4	12.8
TC1-N	58.5 - 60		63.2												
TC1-N	68.5 - 70	18			100	95.2	82	71.2	55.1	39.1	28.9	22.2	15.7	13	11.4
TC1-N	73.5 - 75	24			100	94.8	88	79.7	62.6	44	29.7	19.6	11.4	9.1	8
TC1-N	78.5 - 80		45.9												
TC1-N	88.5 - 80	25				100	95.1	86.5	75.6	61.7	50.2	37	20.9	17.7	16.8
TC1-E	9 - 10.5		82.3												
TC1-E	13.5 - 15	21				100	83.1	74.7	68.5	62.1	53.1	43.8	35.3	28	24.8
TC1-E	28.5 - 30		14.5												
TC1-E	53.5 - 55	17			100	95.7	77	63.9	49.7	36.5	28.1	22.3	17.7	15.1	13.6
TC1-E	58.5 - 60	25			100	90.3	88	85.3	78.5	64.4	43	30	20	17	15.3
TC1-E	63.5 - 65		41.6												
TC1-E	73.5 - 75		69.1												
TC1-E	83.5 - 85	22				100	83.5	72.8	59.8	45.4	34.1	22.8	13.3	11.2	10
TC1-E	88.5 - 90	21				100	96.8	89.9	82.4	66.7	48.7	28.9	14.1	12.4	11.7
TC1-E	98.5 - 100	15				100	96.5	89.5	72.9	47.8	34.5	24.8	15.7	13.2	12
TC1-W	13.5 - 15	26				100	85	76.3	68.8	62.2	55.1	46.3	36.4	27.1	22.5
TC1-W	18.5 - 20		91.9												
TC1-W	33.5 - 35		56.5												
TC1-W	48.5 - 50	26				100	94	92.1	88.2	82.8	78.6	75.2	53.1	22.8	15
TC1-W	68.5 - 70	19			100	96	86.2	72.5	56.1	40	29.3	22.3	14.8	12.7	11.5

				Gradation: Percent Passing Sieve											
Boring	**Depth**	**Moisture Content %**	**Carbonate Content %**	**2-inch**	**1-inch**	**3/4-inch**	**3/8-inch**	**#4**	**#10**	**#20**	**#40**	**#60**	**#100**	**#140**	**#200**
TC1-W	73.5 - 75		65												
TC1-W	78.5 - 80	22			100	91.7	90.8	84.3	69.2	50.6	37	24.6	15.1	12.7	11.5
TC1-W	93.5 - 95	22				100	99	96.1	83.9	59.5	39.3	23.3	17	14.9	13.7
TC1-W	98.5 - 100		67.7												
TC1-S	13.3 - 15	21				100	96.3	90.5	83.8	76.2	68.2	56.6	42.9	30	24.9
TC1-S	18.5 - 20		87.1												
TC1-S	33.5 - 35	20		100	88	87.5	87.5	86.4	81.8	68.4	53	33.7	19.2	8.9	6.1
TC1-S	38.5 - 40		40.7												
TC1-S	43.5 - 45		25.3												
TC1-S	48.5 - 50					100	100	96.4	89.9	84.3	80	77	64	33.1	21.2
TC1-S	68.5 - 70	20				100	94.2	79.8	62.9	45.2	33.2	25.1	16.5	14.1	12.8
TC1-S	73.5 - 75		68.9												
TC1-S	78.5 - 85		67.8												
TC1-S	93.5 - 95	23				100	98	92.2	81.6	58.8	38.5	21.6	14.4	13.6	12.3
TC2-E	18.5 - 20		80.6												
TC2-E	28.5 - 30					100	86.4	77.2	69.5	64.1	60.4	56.5	35.5	15.4	10
TC2-E	48.5 - 50					72.1	63.4	52.6	44.8	37.2	32.7	29.1	21.3	11	7.9
TC2-E	78.5 - 80					95	85.7	76.6	62.1	43.7	30.6	21	13.2	10.8	9.6
TC2-N	5.5 - 7					66.1	61.6	53.9	45.6	28.5	14.7	7.2	4.5	3.6	3.2
TC2-N	8.5 - 10					78.7	64.4	34.4	26.6	21.2	18.2	16.4	15	14.2	13.6
TC2-N	11 - 12.5					81.6	47.2	31.3	17.6	11.6	9	7.5	6.4	5.9	5.5
TC2-N	13.5 - 15					77.5	68.2	53.6	41.6	32.6	25	19.2	14.9	12.4	10.8
TC2-N	28.5 - 30					68.6	59	48.6	35.9	29.3	26.2	23.9	18.8	11.9	7.3
TC2-N	33.5 - 35							100	98.9	97.5	96.2	94.3	64	12.2	4.3
TC2-N	38.5 - 40					84.1	80.5	79.4	77.7	76.2	75.1	73.6	58.8	18.6	8.8
TC2-N	58.5 - 60					93.5	83.8	75	64.7	53.4	4104	32	23.3	19	16.2
TC2-N	63.5 - 65					93.6	86.8	76.3	62	46.6	33.6	23.2	14.3	12.2	10.8

				Gradation: Percent Passing Sieve											
Boring	**Depth**	**Moisture Content %**	**Carbonate Content %**	**2-inch**	**1-inch**	**3/4-inch**	**3/8-inch**	**#4**	**#10**	**#20**	**#40**	**#60**	**#100**	**#140**	**#200**
TC2-N	68.5 - 70					81.1	46.5	33.5	23.9	18.5	15	12.2	9.1	7.3	6.4
TC2-S	8.5 - 10					100	99.2	87.9	79.2	68.8	61.2	55.5	50.3	46.4	43
TC2-S	11 - 12.5		89.5												
TC2-S	43.5 - 45					100	93.4	89.3	84.2	82.7	81.5	80.3	67.7	28.5	16.3
TC2-S	53.5 - 55					100	96.6	86	66.3	43.2	31.3	12.7	16.7	11.8	9.6
TC2-S	78.5 - 80					100	91.2	80.1	65.1	47.9	35.4	24	13	10.7	9.3
TC2-S	83.5 - 85					92.1	74.8	60.4	45.8	33.3	25.5	19.2	12.5	10.8	9.7
TC2-W	11 - 12.5		78.6												
TC2-W	33.5 - 35		23.3				100	98.7	94.3	88.5	84.7	79.6	61.2	22.6	14.4
TC2-W	53.5 - 55					97	83.5	60.1	40.4	27.9	22.4	18.3	13.6	10.4	8.9
TC2-W	73.5 - 75					90.1	81.8	70.8	57.8	42.4	30.5	21	12.6	10.2	8.9
TC2-W	83.5 - 85					100	83.3	66.3	52.6	40.5	32.7	26	19	17.3	16.2

Table 4 Test Cell 2 Soil-Bentonite Cutoff Wall Backfill Slurry Testing Results

Sample Station	Specific Gravity	Gradation (Percent Passing)										Hyd. Cond. cm/sec
		3/4-inch	3/8-inch	No. 4	No. 10	No. 20	No. 40	No. 60	No. 100	No. 140	No. 200	
0+70	2.7	100	100	90.1	76.5	63.3	52.7	44.2	38.1	33.4	29.2	7.9E-08
2+70	2.7	100	100	86.1	72.8	60.5	50.8	43.4	37.4	33.2	29.1	1.9E-08
4+80	2.7	100	100	83.5	66.5	54	43.2	39.1	34.1	30.8	27.6	2.4E-08
6+40	2.7	100	100	92	80	68.1	58	49.9	44.4	39.5	36.5	4.0E-09
8+40	2.7	100	100	90.7	77.4	65.2	55.1	47.2	41.8	37.7	33.7	1.4E-08
10+40	2.7	100	99.6	89.4	76.6	64.2	54	46.4	41.2	37	33.3	2.3E-08
12+40	2.7	100	100	89.5	76.1	63.1	52.6	44.9	39.1	35.2	31.7	6.8E-08
14+40	2.7	100	100	89.5	75.6	61.9	51.1	43.2	37.3	33.3	29.5	8.9E-08
16+30	2.7	100	100	89	75.4	62.7	52.3	44	38.2	33.9	29.9	4.0E-08
13+30	2.7	100	100	88.7	74.9	62.4	52.4	44.3	38.5	34	29.7	6.0E-08

Table 5 Major Test Cell Construction Equipment

Contractor	Equipment	Number On-Site (Maximum)
Barnard Construction	Cat 14G Motorgrader	1
Barnard Construction	John Deere 9420 4-Wheel Drive Tractor with Ag Scraper	2
Barnard Construction	John Deere 9320 4-Wheel Drive Tractor with Ag Scraper	1
Barnard Construction	John Deere 9420 4-Wheel Drive Tractor with Discer	1
Barnard Construction	Case STX425 4-Wheel Drive Tractor with Ag Scraper	1
Barnard Construction	Cat D6MLGP Dozer	1
Barnard Construction	Cat D8R Dozer	2
Barnard Construction	Cat D6R Dozer	1
Barnard Construction	Cat D6R XL Dozer	1
Barnard Construction	Cat D5NLGP Dozer	1
Barnard Construction	Cat D6RLGP Dozer	1
Barnard Construction	Cat 740 Articulated Dump Truck #2 (now 1)	8
Barnard Construction	Cat 345B Excavator	1
Barnard Construction	Cat 350 Excavator	1
Barnard Construction	Cat 330B Excavator	1
Barnard Construction	Cat 345B Excavator	1
Barnard Construction	Cat 385B Excavator	1
Barnard Construction	Cat 375 Excavator	1
Barnard Construction	Cat 320CL Excavator	1
Barnard Construction	Cat 345BL Excavator	1
Barnard Construction	Cat CS-563D Smooth Drum Roller	1
Barnard Construction	Cat CS-533E Smooth Drum Roller	1
Barnard Construction	Cat 980G Loader	1
Barnard Construction	Cat 950G Loader	1
Barnard Construction	Cat 988G Loader	1
Barnard Construction	Cat 950F Loader with forks	1
Barnard Construction	Cat TH103 Forklift	1
Barnard Construction	ULG G12-55A Forklift	1
Barnard Construction	Trencor Jetco Rock Trencher	1
Barnard Construction	MEI Belt Conveyor	1
Barnard Construction	MEI Belt Conveyor	1
Barnard Construction	MEI Belt Conveyor	1
Barnard Construction	MEI Belt Conveyor	1
Barnard Construction	Cedarapid Screening Plant	1
Barnard Construction	McLanahan SCRW-019 Auger Type Washer	1
Barnard Construction	Screen Plant Conveyor	1
Barnard Construction	Nordberg LT1213M Impact Rock Crusher	1
Barnard Construction	Nordberg ST171 Screening Plant (mobile track)	1
Barnard Construction	Powerscreen Screening Plant (towed)	1
Barnard Construction	Case 40XT (Bobcat Type) Loader	1
Barnard Construction	Cat 248 Turbo (Bobcat Type) Loader with Power Broom	1
Barnard Construction	Link Belt HTC 8670 Mobile Crane	1
Barnard Construction	Terex T340 Mobile Crane	1

Contractor	Equipment	Number On-Site (Maximum)
Barnard Construction	Tadano ATF 650XL Mobile Crane	1
Nodarse Associates	Deidrich D-50 All-Terrain Drill	1
Nodarse Associates	Deidrich D-50 Turbo All-Terrain Drill	2
Nodarse Associates	Cat 248 (Bobcat Type) Loader with Auger	1
Inquip	Cat TH83 Forklift	1
Inquip	Cat 345BL Excavator with long arm	1
Inquip	Cat 315C Excavator	1
Inquip	Slurry Mixing Plant	1
Inquip	Cat D6RLGP Dozer	1
Angelini Blasting	Gardner-Denver 1000 Drill	1
Angelini Blasting	Ingersol-Rand DM35 Drill (modified with Krupp Hammer)	1

Table 6 Riprap Test Blast Product Gradations

Particle Size Range	Weight (Pounds)	Percent (of that <0.5 feet)
20 By 20-foot Shot Pattern (17072.5 Pounds Total Sample)		
3 Feet Plus	2058	12.05
3 to 2 Feet	3092	18.11
2 to 1.5 Feet	2278	13.34
1 to ½ Foot	5020.1	29.40
½ Foot	1475	27.09
½ Foot Minus	3149.5	
25 By 25-foot Shot Pattern (15749.1 Pounds Total Sample)		
3 Feet Plus	3576	22.70
3 to 2 Feet	3128	19.86
2 to 1.5 Feet	1315.6	8.35
1 to ½ Foot	3173.9	20.15
½ Foot	1293.4	28.92
½ Foot Minus	3261.9	
28 By 28-foot Shot Pattern (17506.6 Pounds Total Sample)		
3 Feet Plus	1815	10.36
3 to 2 Feet	1629	9.31
2 to 1.5 Feet	4090	23.36
1 to ½ Foot	5154	29.44
½ Foot	1477.7	27.52
½ Foot Minus	3340.6	
35 By 35-foot Shot Pattern (20032.4 Pounds Total Sample)		
3 Feet Plus	4327	21.60
3 to 2 Feet	4814	24.03
2 to 1.5 Feet	3415	17.05
1 to ½ Foot	3213.3	16.04
½ Foot	1799.2	21.28
½ Foot Minus	2463.8	

Table 7 Test Cell 1 Construction Equipment Production Hours Summary (02/22/05 to 03/22/05)

Equipment	Strip Muck	Found'n Treat-ment	Stock-pile to Crush	Canal Cast	Select Fill L/H/P	Rand'm Fill L/H/P	Rockfill Cast	Rockfill L/H/P	Drain & Filter	Bed'g & Riprap	Seep Control Piping	Other	Total
	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)
Graders	0	0	0	0	0	0	0	0	0	0	0	6	**6**
Dozers	0	0	22	0	26	0	0	72	0	0	0	16	**136**
Dump Trucks	0	0	0	0	140	98	0	154	4	58	4	6	**464**
Excavators	0	0	50	0	140	40	0	113	0	72	98	36	**549**
Compaction	0	0	0	0	0	0	0	12	0	0	0	5	**17**
Loaders	0	0	74	0	12	0	0	0	16	28	20	91	**241**
Pumps	0	0	0	0	0	0	0	0	0	0	0	4	**4**
Service	0	0	4	0	0	0	0	0	0	0	0	34	**38**
Generators	0	0	0	0	0	0	0	0	0	0	0	89	**77**
Crushing Plant	0	0	312	0	0	0	0	0	0	0	0	0	**312**
Other	0	0	0	0	0	0	0	0	0	0	0	0	**0**

Table 8 Test Cell 2 Construction Equipment Production Hours Summary (02/22/05 to 03/22/05)

Equipment	Strip Muck	Found'n Treat-ment	Stock-pile to Crush	Canal Cast	Select Fill L/H/P	Rand'm Fill L/H/P	Rockfill Cast	Rockfill L/H/P	Drain & Filter	Bedd'g & Riprap	Seep Control Piping	Other	Total
	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)	(Hrs)
Graders	0	0	0	0	0	0	0	0	0	0	0	4	**4**
Dozers	0	0	0	0	85	196	0	0	30	0	0	30	**341**
Dump Trucks	0	0	0	0	223	356	0	0	55	16	0	0	**650**
Excavators	0	0	0	0	112	153	0	22	8	4	16	56	**371**
Compaction	0	0	0	0	37	15	0	0	12	0	0	8	**72**
Loaders	0	0	6	0	24	20	0	8	63	4	10	64	**199**
Pumps	0	0	0	0	0	0	0	0	0	0	0	4	**0**
Service	0	0	8	0	0	0	0	0	0	0	0	14	**22**
Generators	0	0	0	0	0	0	0	0	0	0	0	97	**65**
Crushing Plant	0	0	0	0	0	0	0	0	0	0	0	0	**0**
Other	0	0	0	0	0	0	0	0	0	0	0	0	**0**

Table 9 Construction Rock Quality Testing

Material	**LA Abrasion Loss (Percent)**	**Soundness Loss (Percent)**
Filter	26.5	12.9
Drain	29.1	3.3
Bedding	30.2	5.5

Table 10 Moisture and Density Testing at Time of Breaching

Test cell	Approximate elevation in embankment (ft)	Approximate depth (ft)	Method	Wet density (pcf)	Dry density (pcf)	Moisture content (%)
1	10	12	Nuclear	139.4	120.9	15.3
	10	12	Nuclear	136.4	116.9	16.7
	14	8	Nuclear	133.5	113.4	17.7
	14	8	Nuclear	141.4	119.2	18.6
	18	4	Nuclear	131.1	110.5	18.7
	10	12	Sand cone	132.8	115.2	15.3
	14	8	Sand cone	137.2	116.5	17.8
	18	4	Sand cone	133.3	112.8	18.2
2	10	12	Nuclear	134.1	117.8	13.8
	10	12	Nuclear	134.1	114.3	17.3
	14	8	Nuclear	133.5	115.9	15.1
	14	8	Nuclear	130.6	112.4	16.2
	18	4	Nuclear	136.8	117.4	16.5
	18	4	Nuclear	135.1	114.1	18.5
	20	2	Nuclear	128.7	109.9	17.1
	20	2	Nuclear	133.8	113.6	17.8
	10	12	Sand cone	136.1	118.1	15.3
	14	8	Sand cone	133.9	117.6	13.9
	18	4	Sand cone	137.6	117.1	17.5
	20	2	Sand cone	132.5	112.6	17.7

FIGURES

Figure 2 Modified Test Cell 1 Section

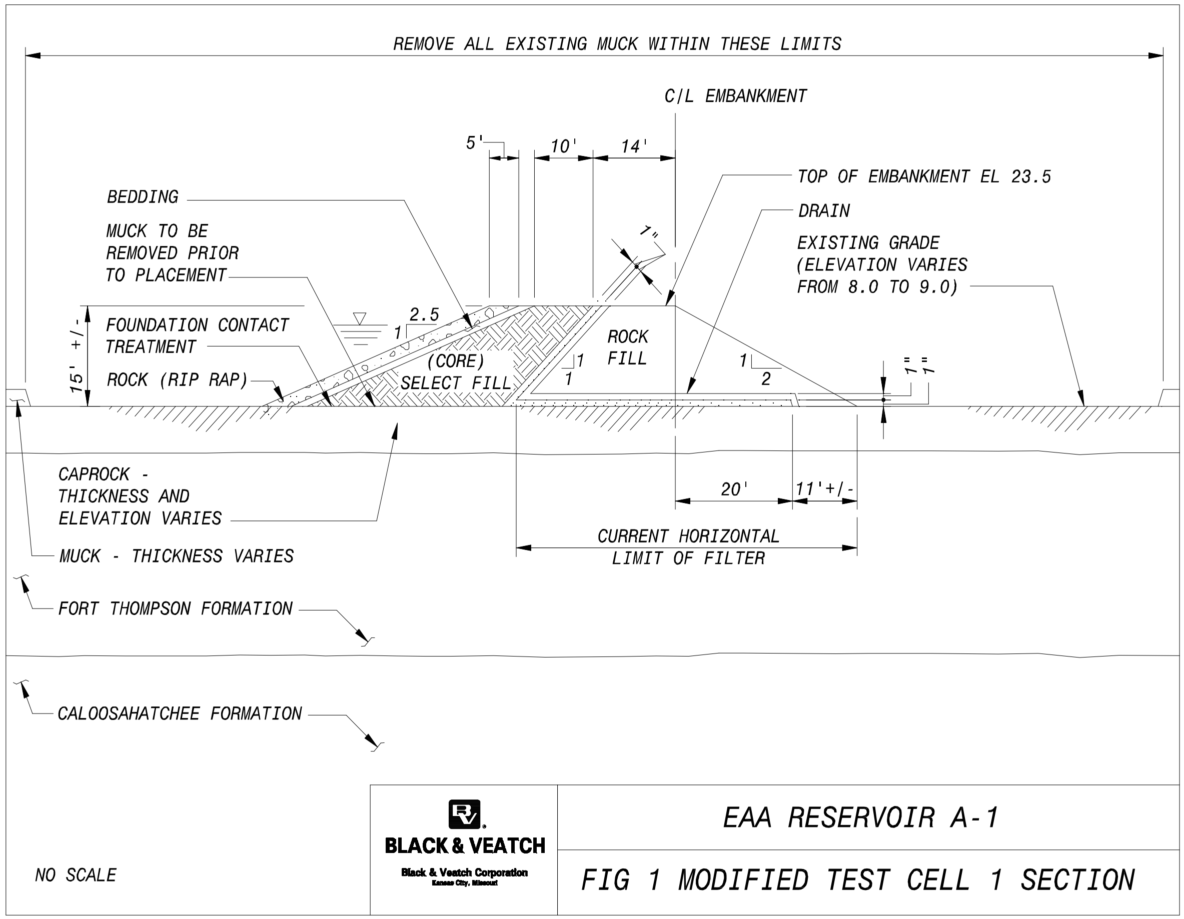

Figure 3 Select Fill Farm Moisture Data Graph

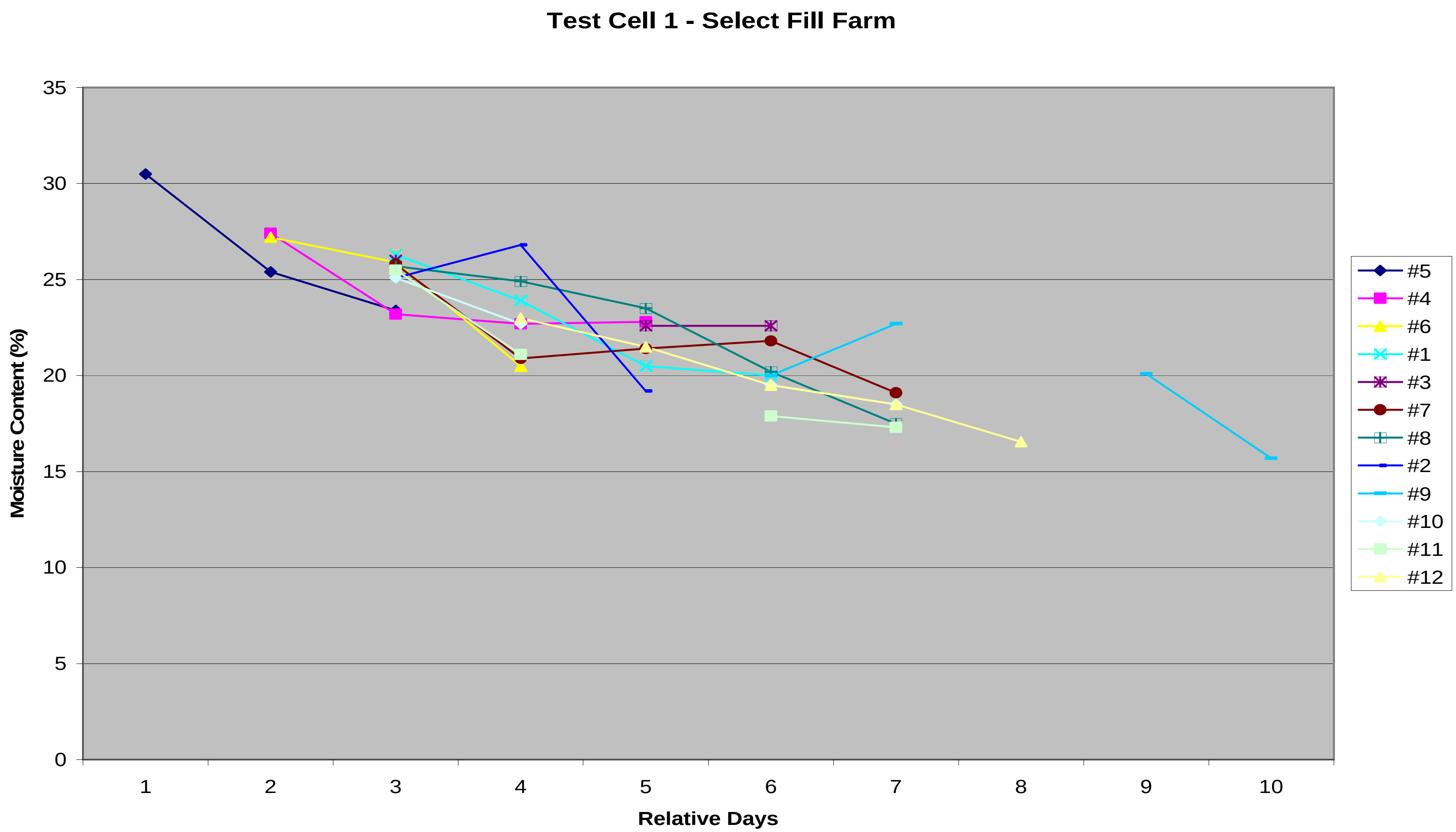

Figure 4 Test Blast Riprap Gradation